Notice No.9

Rules and Regulations for the Classification of Special Service Craft, July 2022

The status of this Rule set is amended as shown and is now to be read in conjunction with this and prior Notices. Any corrigenda included in the Notice are effective immediately.

Please note that corrigenda amends to paragraphs, Tables and Figures are not shown in their entirety.

Issue date: June 2023

Amendments to	**Adoption Date**	**Applicable Date**
Part 3, Chapter 4, Section 5	Corrigenda	N/A
Part 6, Chapter 3, Section 3	Corrigenda	N/A
Part 6, Chapter 7, Section 4	Corrigendum	N/A
Part 7, Chapter 7, Section 4	Corrigendum	N/A

Part 3, Chapter 4
Closing Arrangements and Outfit

■ *Section 5*
Hatches on exposed decks

5.6 Structural details

5.6.1 Various structural details for hatchways and access openings are given in ~~the LR *Guidance Notes for Structural Details*~~ LR's *Guidance Notes for the Classification of Special Service Craft – Version 1.0 Design Details***.**

All instances of the above amendment have been changed throughout this Ruleset.

Part 6, Chapter 3
Scantling Determination for Mono-Hull Craft

■ *Section 3*
Shell envelope plating

3.2 Plate keel

3.2.1 The breadth b k, and thickness t k, of the plate are not to be taken as less than:

~~$t_k = \sqrt{k_s}\, 1,35L\ {}_{0,45}{}_R\ \text{mm}$~~ $t_k = \sqrt{k_s} \times 1{,}35 L_R^{0{,}45} mm$

Part 6, Chapter 7
Failure Modes Control

■ *Section 4*
Buckling Control

4.4 Derivation of the buckling stress for plate panels

Table 7.4.2 Plate panel buckling requirements

	Stress field	Buckling interaction formula	
(e)	Bi-axial compressive loads plus shear loads	~~$\dfrac{\mathbf{0,625}\left(\mathbf{1}+\dfrac{\mathbf{0,6}}{A_\mathbf{R}}\right)\left(\dfrac{\sigma_\mathbf{dy}}{\sigma_\mathbf{cy}}\right)}{\left(\mathbf{1}-\mathbf{0,625}\right)\left(\dfrac{\sigma_\mathbf{dx}}{\sigma_\mathbf{cx}}\right)}+\dfrac{\left(\dfrac{\tau_\mathbf{d}}{\tau_\mathbf{c}}\right)^\mathbf{2}}{\mathbf{1}-\left(\dfrac{\sigma_\mathbf{dx}}{\sigma_\mathbf{cx}}\right)}\le\mathbf{1}$~~ $\mathbf{0,625}\,\dfrac{\left(\mathbf{1}+\dfrac{\mathbf{0,6}}{A_R}\right)\left(\dfrac{\sigma_\mathbf{dy}}{\sigma_\mathbf{cy}}\right)}{\mathbf{1}-\mathbf{0,625}\left(\dfrac{\sigma_\mathbf{dx}}{\sigma_\mathbf{cx}}\right)}+\dfrac{\left(\dfrac{\tau_\mathbf{d}}{\tau_\mathbf{c}}\right)^2}{\mathbf{1}-\left(\dfrac{\sigma_\mathbf{dx}}{\sigma_\mathbf{cx}}\right)}\le\mathbf{1}$	(diagram not shown)

4.5 Additional requirements for plate panels which buckle elastically

Table 7.4.3 Buckling stress of secondary stiffeners

Mode	Elastic buckling stress, N/mm^2	Critical buckling stress, N/mm^2 *see* Note 1
(a) Overall buckling (perpendicular to plane of plating without rotation of cross-section)	~~$\sigma_{e(a)} = C_f 0,001E \frac{I_a}{A_{te} l_{e2}}$~~ $\sigma_{e(a)} = C_f 0,001E \frac{I_a}{A_{te} l_e^2}$	$\sigma_{c(a)}$
(b) Torsional buckling	~~$\sigma_{e(t)} = \frac{0,001EI_w}{I_p l_{e2}}\left(m2 + \frac{K}{m2}\right) + 0,385E\frac{I_t}{I_p}$~~ $\sigma_{e(t)} = \frac{0,001EI_w}{I_p l_e^2}\left(m^2 + \frac{K}{m^2}\right) + 0,385E\frac{I_t}{I_p}$	$\sigma_{c(t)}$
(c) Web buckling (excluding flat bar stiffeners)	~~$\sigma_{e(w)} = 3,8E\left(\frac{t_w}{d_w}\right)2$~~ $\sigma_{e(w)} = 3,8E\left(\frac{t_w}{d_w}\right)^2$	$\sigma_{c(w)}$
(d) Flange buckling	~~$\sigma_{e(f)} = 3,8E\left(\frac{t_f}{b_f}\right)2$~~ $\sigma_{e(f)} = 0,39E\left(\frac{t_f}{b_f}\right)^2$	$\sigma_{c(f)}$

NOTE

Symbols

~~I_t~~ I_t = St.Venant's torsional moment of inertia, in cm^4, of longitudinal (without attached plating)

~~$= \frac{d_w t_{w3}}{3} 10^{-4}$~~ for flat bars

$= \frac{d_w t_w^3}{3} 10^{-4}$ for flat bars

~~$= \frac{1}{3}\left[d_w t_{w3} + b_f t_{f3}\left(1 - \frac{0,63t_f}{b_f}\right)\right] 10^{-4}$~~ for built up profiles, rolled angles and bulb plates

$= \frac{1}{3}\left[d_w t_w^3 + b_f t_f^3\left(1 - \frac{0,63t_f}{b_f}\right)\right] 10^{-4}$ for built up profiles, rolled angles and bulb plates

~~I_p~~ I_p = polar moment of inertia, in cm^4, of profile about connection of stiffener to plating

$= \frac{d_w^3 t_w}{3} 10^{-4}$ for flat bars

~~$= \left(\frac{d_{w3} t_w}{3} + d_{w2} b_f t_f\right) 10^{-4}$~~ for built up profiles, rolled angles and bulb plates

$= \left(\frac{d_w^3 t_w}{3} + d_w^2 b_f t_f\right) 10^{-4}$ for built up profiles, rolled angles and bulb plates

~~I_w~~ I_w = sectional moment of inertia, in cm^6, of profile about connection of stiffener to plating

~~$= \frac{d_{w3} t_{w3}}{36} 10^{-6}$~~ for flat bars

$= \frac{d_w^3 t_w^3}{36} 10^{-6}$ for flat bars

~~$= \frac{t_f b_{f3} d_{w2}}{12} 10^{-6}$~~ for ‘Tee’ profiles

$= \frac{t_f b_f^3 d_w^2}{12} 10^{-6}$ for ‘Tee’ profiles

~~$= \frac{b_{f3} d_{w3}}{12(b_f + d_w)_2}\left(t_f\left(b_f^2 + 2b_f d_w + 4{d_w}^2\right) + 3t_w b_f d_w\right)10^{-6}$~~ for ‘L’ profiles, rolled angles and bulb plates

$= \frac{b_f^3 d_w^2}{12(b_f + d_w)^2}\left(t_f\left(b_f^2 + 2b_f d_w + 4d_w^2\right) + 3t_w b_f d_w\right)10^{-6}$ for ‘L’ profiles, rolled angles and bulb plates

C = spring stiffness exerted by supporting plate panel

~~$= \frac{k_p E t_{p3}}{3b\left(1 + \frac{1,33 k_p d_w t_{p3}}{b t_{w3}}\right)}$~~

$= \frac{k_p E t_p^3}{3b\left(1 + \frac{1,33 k_p d_w t_p^3}{b t_w^3}\right)}$

K ~~$= \frac{\mathbf{1{,}03}\boldsymbol{CS_4}}{\boldsymbol{EI_w}}\mathbf{10^4}$~~
$= \frac{\mathbf{1{,}03}\boldsymbol{CS^4}}{\boldsymbol{EI_w}}\mathbf{10^4}$

4.5.4 The ultimate buckling strength of plating, σ_u, which buckles elastically, may be determined as follows:

(a) shortest edge loaded, i.e. $A_R \geq 1$:

~~$\sigma_u = \sigma_0\left(\frac{1{,}9}{W} - \frac{0{,}8}{W2}\right)\mathrm{N/mm}^2$~~

$$\sigma_u = \sigma_0\left(\frac{1{,}9}{W} - \frac{0{,}8}{W^2}\right)\mathrm{N/mm}^2$$

~~(a)~~(b) longest edge loaded, i.e. $A_R < 1$:

~~$\sigma_u = \frac{1{,}77\sigma_0 A_{0{,}78R}}{W}\mathrm{N/mm}^2$~~

$$\sigma_u = \frac{1{,}77\sigma_0 A_R^{0{,}78}}{W}\mathrm{N/mm}^2$$

4.6 Shear buckling of stiffened panels

4.6.2 The elastic shear buckling stress of longitudinally stiffened panels between primary members may be taken as:
~~$\tau_e = K_{\ s}E\left(\frac{t}{s}\right)2$ for A R ≥ 1~~
$\tau_e = K_s E\left(\frac{t}{s}\right)^2$ for $A_R \geq 1$
where
~~K_S~~ K_S = ~~$4{,}5\left(\left(\frac{s}{1000l}\right)2 + \frac{1}{N2} + \left(\frac{N2 - l}{N2}\right)\left(\frac{\omega}{1+\omega}\right)r\right)$~~
$4{,}5\left(\left(\frac{s}{1000l}\right)^2 + \frac{1}{N^2} + \left(\frac{N^2 - 1}{N^2}\right)\left(\frac{\omega}{1+\omega}\right)^r\right)$
ω = ~~$\frac{\boldsymbol{l0I_{se}}}{\boldsymbol{lt3}}$~~
$\frac{\mathbf{10}\boldsymbol{I_{se}}}{\boldsymbol{lt^3}}$

…..

4.8 Secondary stiffening perpendicular to direction of compression

4.8.2 The minimum moment of inertia of each stiffener including attached plating of width, *s*, to ensure that overall panel buckling does not precede plate buckling is to be taken as:

~~$\boldsymbol{I_s = \frac{Ds(4N_{L2} - 1)((N_{L2} - 1)_2 - 2(N_{L2} + 1)\kappa + \kappa_2)}{2(5N_{L2} + 1 - \kappa)\Pi_4 E}\mathrm{mm4}}$~~

$$\boldsymbol{I_s = \frac{Ds(4N_L^2 - 1)\left((N_L^2 - 1)^2 - 2(N_L^2 + 1)\kappa + \kappa^2\right)}{2(5N_L^2 + 1 - \kappa)\Pi^4 E}\mathrm{mm}^4}$$

where
D = ~~$\frac{Et_{p3}}{12(1 - \nu_2)}$~~
$\frac{Et_p^3}{12(1 - \nu^2)}$

4.9 Buckling of primary members

4.9.2 To prevent global buckling from occurring before local panel buckling, transverse primary girders supporting axially loaded longitudinal stiffeners are to have a sectional moment of inertia, including attached plating, of not less than the following:

~~$$I_g = \frac{0{,}35S_{p4}I_s}{I_{3s}} X\ 10_3 \mathrm{cm}_4$$~~

$$I_g = \frac{0{,}35S_p^4 I_s}{I_S^3} 10^3 \mathrm{cm}^4$$

~~S_p~~ S_p and s are as defined in *Pt 6, Ch 7, 4.2 Symbols 4.2.1*, *see also Figure 7.4.1 Shear buckling of stiffened panels=*

~~I_g~~ I_g = *is the sectional moment of inertia including attached plating*

I_s = moment of inertia of secondary stiffeners, in cm^4, required to satisfy the elastic column buckling mode specified in *Table 7.4.3 Buckling stress of secondary stiffeners*

= ~~$\frac{\sigma_{ep}A_{te}l_{e2}}{0{,}001E}$~~

$\frac{\sigma_{ep}A_{te}l_e^2}{0{,}001E}$

where

σ_{ep} = $1{,}2\sigma_d$ N/mm^2 for $\sigma_{e(a)}$ < ~~$\frac{\sigma_0}{2}$~~ $< \frac{\sigma_0}{2}$

~~$\frac{\sigma_{20}}{4(\sigma_0 - 1{,}2\sigma_d)}$ for $\sigma_{e(a)} \geq \frac{\sigma_0}{2}$~~

$\frac{\sigma_0^2}{4(\sigma_0 - 1{,}2\sigma_d)}$ for $\sigma_{e(a)} \geq \frac{\sigma_0}{2}$

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4.10 Shear buckling of girder webs

4.10.2 The critical shear buckling stress, τ_c, is to be determined using the following formula for τ_ε and Note 1 in *Table 7.4.1 Buckling stress of plate panels*.

τ_e = ~~$3{,}62\left(1{,}335 + \left(\frac{d_w}{1000l_p}\right)2\right)E\left(\frac{t_w}{d_w}\right)2$ N/mm^2~~

$$3{,}62\left(1{,}335 + \left(\frac{d_w}{1000l_p}\right)^2\right)E\left(\frac{t_w}{d_w}\right)^2 \text{ N/mm}^2$$

Part 7, Chapter 7
Failure Modes Control

■ *Section 4*
Buckling Control

4.1 General

4.1.3 In general all areas of the structure are to meet the buckling strength requirements for the design stresses. The design stresses are to be taken as follows:

(a) Global hull girder bending and shear stresses given in ~~Chapter 6~~ *Pt 7, Ch 6 Hull Girder Strength*, but not including stresses ~~σ_l~~ σ_l and ~~σ_t~~ σ_t as defined in *Table 6.2.1 Longitudinal component stresses* in ~~*Pt 6, Ch 6 Hull Girder Strength*~~ *Pt 7, Ch 6 Hull Girder Strength*.

4.4 Derivation of the buckling stress of plate panels

Table 7.4.1 Buckling stress of plate panels

Mode	Elastic buckling stress, N/mm^2, *see* Note	
(a) Uni-axial compression: ~~*(i)*~~ (i) Long narrow panels, loaded on the narrow edge ~~*(ii)*~~ (ii) Short broad panels, loaded on the broad edge	~~A R~~ A$_R$ ≥ 1 ~~$\sigma_e = 3,62\varphi E\left(\frac{t_p}{b}\right)2$~~ $\sigma_e = 3,62\varphi E\left(\frac{t_p}{b}\right)^2$ ~~A R~~ A$_R$ ≥ 1 ~~$\sigma_e = 0,9C\varphi\left(\frac{b}{a}+\frac{a}{b}\right)2E\left(\frac{t_p}{b}\right)2$~~ $\sigma_e = 0,9C\varphi\left(\frac{b}{a}+\frac{a}{b}\right)^2 E\left(\frac{t_p}{b}\right)^2$	(diagram not shown)
(b) Pure shear:	~~$\tau_e = 3,62\left(1,335+\left(\frac{u}{v}\right)2\right)E\left(\frac{t_p}{u}\right)2$~~ $\tau_e = 3,62\left(1,335+\left(\frac{u}{v}\right)^2\right)E\left(\frac{t_p}{u}\right)^2$	(diagram not shown)

Table 7.4.2 Plate panel buckling requirements

	Stress field	Buckling interaction formula	
(d)	Uni-axial compressive loads plus shear load	for ~~A R~~ A$_R$ > 1 ~~$\left(\frac{\sigma_d}{\sigma_c}\right)+\left(\frac{\tau_d}{\tau_c}\right)2 \le 1$~~ $\left(\frac{\sigma_d}{\sigma_c}\right)+\left(\frac{\tau_d}{\tau_c}\right)^2 \le 1$ for ~~A R~~ A$_R$ > 1 ~~$\left(\frac{1+0,6A_R}{1,6}\right)\left(\frac{\sigma_d}{\sigma_c}\right)+\left(\frac{\tau_d}{\tau_c}\right)2 \le 1$~~ $\left(\frac{1+0,6A_R}{1,6}\right)\left(\frac{\sigma_d}{\sigma_c}\right)+\left(\frac{\tau_d}{\tau_c}\right)^2 \le 1$	(diagram not shown)
(e)	Bi-axial compressive loads plus shear loads	~~$\frac{0,625\left(1+\frac{0,6}{A_R}\right)\left(\frac{\sigma_{dy}}{\sigma_{cy}}\right)}{1-0,625\left(\frac{\sigma_{dx}}{\sigma_{cx}}\right)}+\frac{\left(\frac{\tau_d}{\tau_c}\right)2=}{1-\left(\frac{\sigma_{dx}}{\sigma_{cx}}\right)} \le 1$~~ $0,625\frac{\left(1+\frac{0,6}{A_R}\right)\left(\frac{\sigma_{dy}}{\sigma_{cy}}\right)}{1-0,625\left(\frac{\sigma_{dx}}{\sigma_{cx}}\right)}+\frac{\left(\frac{\tau_d}{\tau_c}\right)^2}{1-\left(\frac{\sigma_{dx}}{\sigma_{cx}}\right)} \le 1$	(diagram not shown)

4.5 Additional requirements for plate panels which buckle elastically

Table 7.4.3 Buckling stress of secondary stiffeners

Mode	Elastic buckling stress, N/mm^2	Critical buckling stress, N/mm^2 *see* Note 1
(a) Overall buckling (perpendicular to plane of plating without rotation of cross-section)	~~$\sigma_{e(a)} = C_f 0,001E\frac{I_a}{A_{te}l_{e2}}$~~ $\sigma_{e(a)} = C_f 0,001E\frac{I_a}{A_{te}l_e^2}$	$\sigma_{c(a)}$
(b) Torsional buckling	~~$\sigma_{e(t)} = \frac{0,001EI_w}{I_p l_{e2}}\left(m2+\frac{K}{m2}\right)+0,385E\frac{I_t}{I_p}$~~ $\sigma_{e(t)} = \frac{0,001EI_w}{I_p l_e^2}\left(m^2+\frac{K}{m^2}\right)+0,385E\frac{I_t}{I_p}$	$\sigma_{c(t)}$

(c) Web buckling (excluding flat bar stiffeners)		~~$\sigma_{\mathrm{e(w)}} = 3,8E\left(\frac{t_{\mathrm{w}}}{d_{\mathrm{w}}}\right)2$~~ $\sigma_{\mathrm{e(w)}} = 3,8E\left(\frac{t_{\mathrm{w}}}{d_{\mathrm{w}}}\right)^2$	$\sigma_{\mathrm{c(w)}}$
(d) Flange buckling		~~$\sigma_{\mathrm{e(f)}} = 0,39E\left(\frac{t_{\mathrm{f}}}{b_{\mathrm{f}}}\right)2$~~ $\sigma_{\mathrm{e(f)}} = 0,39E\left(\frac{t_{\mathrm{f}}}{b_{\mathrm{f}}}\right)^2$	$\sigma_{\mathrm{c(f)}}$
Symbols			
~~I_{t}~~ I_{t}	= St.Venant's torsional moment of inertia, in cm^4, of longitudinal (without attached plating) ~~$= \frac{d_{\mathrm{w}}t_{\mathrm{w3}}}{3}10^{-4}$~~ for flat bars $= \frac{d_{\mathrm{w}}t_{\mathrm{w}}^3}{3}10^{-4}$ for flat bars ~~$= \frac{1}{3}\left[d_{\mathrm{w}}t_{\mathrm{w3}} + b_{\mathrm{f}}t_{\mathrm{f3}}\left(1-\frac{0,63t_{\mathrm{f}}}{b_{\mathrm{f}}}\right)\right]10^{-4}$~~ for built up profiles, rolled angles and bulb plates $= \frac{1}{3}\left[d_{\mathrm{w}}t_{\mathrm{w}}^3 + b_{\mathrm{f}}t_{\mathrm{f}}^3\left(1-\frac{0,63t_{\mathrm{f}}}{b_{\mathrm{f}}}\right)\right]10^{-4}$ for built up profiles, rolled angles and bulb plates		
~~I_{p}~~ I_{p}	= polar moment of inertia, in cm^4, of profile about connection of stiffener to plating ~~$= \frac{d_{\mathrm{w}}t_{\mathrm{w3}}}{3}10^{-4}$~~ for flat bars $= \frac{d_{\mathrm{w}}^3t_{\mathrm{w}}}{3}10^{-4}$ for flat bars ~~$= \left(\frac{d_{\mathrm{w3}}t_{\mathrm{w}}}{3} + d_{\mathrm{w2}}b_{\mathrm{f}}t_{\mathrm{f}}\right)10^{-4}$~~ for built up profiles, rolled angles and bulb plates $= \left(\frac{d_{\mathrm{w}}^3t_{\mathrm{w}}}{3} + d_{\mathrm{w}}^2b_{\mathrm{f}}t_{\mathrm{f}}\right)10^{-4}$ for built up profiles, rolled angles and bulb plates		
~~I_{w}~~ I_{w}	= sectorial moment of inertia, in cm^6, of profile about connection of stiffener to plating ~~$= \frac{d_{\mathrm{w3}}t_{\mathrm{w3}}}{36}10^{-6}$~~ for flat bars $= \frac{d_{\mathrm{w}}^3t_{\mathrm{w}}^3}{36}10^{-6}$ for flat bars ~~$= \frac{t_{\mathrm{f}}b_{\mathrm{f3}}d_{\mathrm{w2}}}{12}10^{-6}$~~ for 'Tee' profiles $= \frac{t_{\mathrm{f}}b_{\mathrm{f}}^3d_{\mathrm{w}}^2}{12}10^{-6}$ for 'Tee' profiles ~~$= \frac{b_{\mathrm{f3}}d_{\mathrm{w3}}}{12(b_{\mathrm{f}}+d_{\mathrm{w}})2}\left(t_{\mathrm{f}}\left(b_{\mathrm{f}}^2 + 2b_{\mathrm{f}}d_{\mathrm{w}} + 4d_{\mathrm{w}}{}^2\right) + 3t_{\mathrm{w}}b_{\mathrm{f}}d_{\mathrm{w}}\right)10^{-6}$~~ for 'L' profiles, rolled angles and bulb plates $= \frac{b_{\mathrm{f}}^3d_{\mathrm{w}}^2}{12(b_{\mathrm{f}}+d_{\mathrm{w}})^2}\left(t_{\mathrm{f}}\left(b_{\mathrm{f}}^2 + 2b_{\mathrm{f}}d_{\mathrm{w}} + 4d_{\mathrm{w}}^2\right) + 3t_{\mathrm{w}}b_{\mathrm{f}}d_{\mathrm{w}}\right)10^{-6}$ for 'L' profiles, rolled angles and bulb plates		
C	= spring stiffness exerted by supporting plate panel ~~$= \frac{k_{\mathrm{p}}Et_{\mathrm{p3}}}{3b\left(1+\frac{1,33k_{\mathrm{p}}d_{\mathrm{w}}t_{\mathrm{p3}}}{bt_{\mathrm{w3}}}\right)}$~~ $= \frac{k_{\mathrm{p}}Et_{\mathrm{p}}^3}{3b\left(1+\frac{1,33k_{\mathrm{p}}d_{\mathrm{w}}t_{\mathrm{p}}^3}{bt_{\mathrm{w}}^3}\right)}$		
K	~~$= \frac{1,03CS_4}{EI_{\mathrm{w}}}10_4$~~ $= \frac{1,03CS^4}{EI_{\mathrm{w}}}10^4$		

4.5 Additional requirements for plate panels which buckle elastically

4.5.4 The ultimate buckling strength of plating, σ_{u}, which buckles elastically, may be determined as follows:

(a) shortest edge loaded, i.e. ~~A_{R}~~ $A_{\mathrm{R}} \geq 1$:

~~(b) $\sigma_{\mathrm{u}} = \sigma_0\left(\frac{1,9}{W} - \frac{0,8}{W2}\right)$ N/mm2~~

$$\sigma_u = \sigma_0 \left(\frac{1{,}9}{W} - \frac{0{,}8}{W^2} \right) \text{N/mm}^2$$

~~(c)~~ (b) longest edge loaded i.e. ~~A_R~~ A_R < 1:

~~(d) $\sigma_u = \frac{1{,}77\sigma_0 A_{0{,}78R}}{W} \text{N/mm}^2$~~

$$\sigma_u = \frac{1{,}77\sigma_0 A_R^{0{,}78}}{W} \text{N/mm}^2$$

4.6 Shear buckling of stiffened panels

4.6.2 The elastic shear buckling stress of longitudinally stiffened panels between primary members may be taken as:

~~$\tau_e = K_s E \left(\frac{t}{s}\right) 2$ for $A_R \geq 1$~~

$\tau_e = K_s E \left(\frac{t}{s}\right)^2$ for $A_R \geq 1$

where

~~K_S~~ K_S = ~~$4{,}5\left(\left(\frac{s}{1000l}\right)2 + \frac{1}{N2} + \left(\frac{N2-l}{N2}\right)\left(\frac{\omega}{1+\omega}\right)r\right)$~~

$$4{,}5\left(\left(\frac{s}{1000l}\right)^2 + \frac{1}{N^2} + \left(\frac{N^2-1}{N^2}\right)\left(\frac{\omega}{1+\omega}\right)^r\right)$$

ω = ~~$\frac{l0I_{se}}{lt3}$~~

$$\frac{10I_{se}}{lt^3}$$

…..

4.8 Secondary stiffening perpendicular to direction of compression

4.8.2 The minimum moment of inertia of each stiffener including attached plating of width, *s*, to ensure that overall panel buckling does not precede plate buckling is to be taken as:

~~$I_s = \frac{Ds(4N_{L2}-1)((N_{L2}-1)_2 - 2(N_{L2}+1)\kappa + \kappa_2)}{2(5N_{L2}+1-\kappa)\Pi_4 E} \text{mm4}$~~

$$I_s = \frac{Ds(4N_L^2-1)\left((N_L^2-1)^2 - 2(N_L^2+1)\kappa + \kappa^2\right)}{2(5N_L^2+1-\kappa)\Pi^4 E} \text{mm}^4$$

where

D = ~~$\frac{Et_{p3}}{12(1-\nu_2)}$~~

$$\frac{Et_p^3}{12(1-\nu^2)}$$

4.9 Buckling of primary members

4.9.2 To prevent global buckling from occurring before local panel buckling, transverse primary girders supporting axially loaded longitudinal stiffeners are to have a sectional moment of inertia, including attached plating, of not less than the following:

~~$I_g = \frac{0{,}35 S_{p4} I_s}{I_{3S}} X\ 10_3\ cm_4$~~

$$I_g = \frac{0{,}35 S_p^4 I_s}{I_S^3} 10^3 \text{cm}^4$$

~~S_p~~ S_p and s are as defined in *Pt 6, Ch 7, 4.2 Symbols 4.2.1*, *see also Figure 7.4.1 Shear buckling of stiffened panels*

~~I_g~~ I_g = is the sectional moment of inertia including attached plating

~~I_s~~ I_s = moment of inertia of secondary stiffeners, in cm^4, required to satisfy the elastic column buckling mode specified in *Table 7.4.3 Buckling stress of secondary stiffeners*

= ~~$\frac{\sigma_{ep} A_{te} l_{e2}}{0{,}001E}$~~

$$\frac{\sigma_{ep} A_{te} l_e^2}{0{,}001E}$$

where

σ_{ep} = $1{,}2\sigma_d$ N/mm^2 for $\sigma_{e(a)} < \frac{\sigma_0}{2}$ $\frac{\sigma_0}{2}$

~~$\frac{\sigma_{20}}{4(\sigma_0 - 1{,}2\sigma_d)}$ $for\ \sigma_{e(a)} \geq \frac{\sigma_0}{2}$~~

$$\frac{\sigma_0^2}{4(\sigma_0 - 1{,}2\sigma_d)}\ for\ \sigma_{e(a)} \geq \frac{\sigma_0}{2}$$

4.10 Shear buckling of girder webs

4.10.2 The critical shear buckling stress, τ_c, is to be determined using the following formula for τ_ε and Note 1 in *Table 7.4.1 Buckling stress of plate panels*.

τ_e = ~~$3{,}62\left(1{,}335 + \left(\frac{d_w}{1000 l_p}\right) 2\right) E \left(\frac{t_w}{d_w}\right) 2$ N/mm^2~~

$$3{,}62\left(1{,}335 + \left(\frac{d_w}{1000 l_p}\right)^2\right) E \left(\frac{t_w}{d_w}\right)^2 \text{ N/mm}^2$$

4.11 Pillars and pillar bulkheads

Figure 7.4.3 Interaction limiting stress curves of G for plate panels subject to bi-axial compression, *see* ~~*Table 7.4.2 Plate panel buckling requirements*~~ *Pt 7, Ch 7, 4.4 Derivation of the buckling stress for plate panels 4.4.2*

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Registered office (Reg. no. 08126909)
71 Fenchurch Street, London, EC3M 4BS
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